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NUMERICAL MODELING OF SEISMIC SOIL-PILE-SUPERSTRUCTURE INTERACTION

Thomas M H LOK[1], Juan M PESTANA[2] and Raymond B SEED[3]

SUMMARY

The prediction of the performance of a structure supported on pile foundations is of paramount importance in the seismic design and assessment of new and existing structures. This paper presents the key elements of a hybrid Finite Element Technique to perform the coupled soil-pile-superstructure and site response analysis. Model simulations of a series of large scale seismic soil-pile-superstructure tests conducted on the 6.1m x 6.1m UC Berkeley/PEER center multi-directional shaking table are used to calibrate and verify the proposed numerical technique. Detailed comparison with measured structural and 'free field' soil response in the shaking table tests show that the proposed formulation gives very good descriptions of maximum spectral accelerations and frequency response. The model accurately captures the transition from kinematic and inertial response of these 'simple' systems.

INTRODUCTION

The prediction of the performance of structures supported on pile foundations is of paramount importance in the seismic design and assessment of existing structures, such as bridges, and their foundation system. Currently, the complex seismic soil-pile-structure interaction is typically evaluated by considering the "fundamental" structural seismic response and then driving this response onto the foundation system. The main shortcoming of this type of analysis is perhaps the fact that the isolated "fundamental" structural behavior does not capture appropriately the response of the foundation system, specially for cases in which significant nonlinearity in the foundation soil is expected [Abghari and Chai, 1995].

There has been a significant amount of research on the lateral response of single piles and pile groups under dynamic loading. The analysis of this complex soil-structure interaction problem has involved the use of finite element analyses [e.g., Kuhlemeyer, 1978], boundary element analyses [e.g., Kaynia and Kausel, 1982] and the Nonlinear Winkler model [e.g., Matlock et al., 1978]. Matlock et al. (1978) first implemented the “Beam on Non-linear Winkler Foundation” (BNLWF) model in a computer code called SPASM to perform dynamic analysis of off-shore structures. Subsequently, numerous researchers have studied the seismic soil-pile-super-structure problem using this technique [e.g., Kagawa, 1983; Nogami et al., 1992]. This model is composed of a linear elastic beam-column representing the pile and non-linear p-y springs and dashpots representing the surrounding soil (cf., Figure 1). The "free field" describes the site response in the absence of the structure. For the uncoupled analysis, the solution proceeds in two steps, namely, the computation of the free field motions and the analysis of the pile-superstructure response. The free field motions, which are a major input to the analysis, are calculated separately through a site response analysis using computer codes such as SHAKE [Schnabel et al., 1972]. The motions are then used in the second stage as input boundary conditions. In the coupled formulation, the site response analysis and the soil-pile-superstructure interaction problem are solved simultaneously in a single step with only the input boundary condition at the lower portion of the soil profile as shown in Figure 1 [Lok and Pestana, 1996; Lok et al., 1998]. The following paragraphs briefly summarize the key elements of the coupled formulation and illustrates the capabilities by comparing with the large shaking table tests results of single piles in soft clay.

[1] *Lecturer, Faculty of Science & Technology, University of Macao*
[2] *Assistant Professor, Depart. Civil & Envir. Engng., University of California, Berkeley*
[3] *Professor, Depart. Civil & Envir. Engng., University of California, Berkeley*

NUMERICAL MODELING

In this study, a hybrid formulation incorporates the Beam on Nonlinear Winkler Foundation (BNWF) model in a dynamic finite element computer code, GeoFEAP. This program is based on the original program, FEAP [Taylor, 1998] with additional constitutive models for geotechnical materials, and has been used extensively for instruction and research [Bray et al., 1995]. The BNWF is composed of a linear elastic beam-column representing the pile as well as the column elements and nonlinear p-y springs and dashpots representing the surrounding "near field" soil. The near field soil response is very complex and its numerical modeling must include the 'closest' but at the same time numerically efficient description of gapping which is observed during large lateral displacements of piles in cohesive soils. The element describing the p-y response includes a nonlinear spring (or a number of linear springs in parallel, in earlier formulations) with gapping as well as residual shear. Lok [1999] implemented new 2-D solid elements incorporating the equivalent linear method for site response analyses and a new enhanced hysteretic constitutive law for nonlinear analyses. This allows GeoFEAP to perform the seismic soil-pile-structure interaction analyses, while simultaneously determining the 1-D site response.

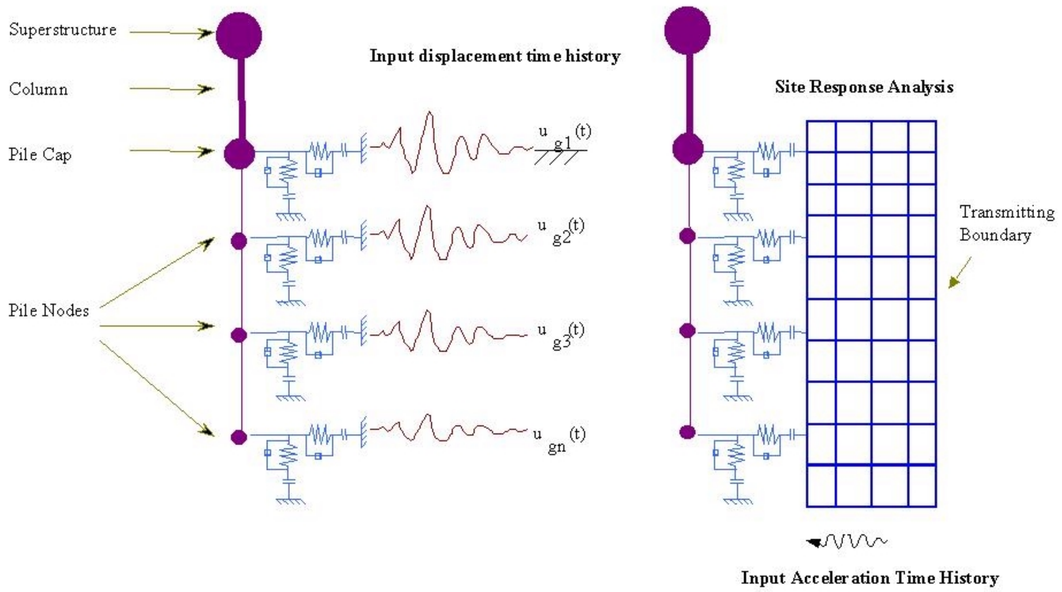


Figure 1: Uncoupled and Coupled Seismic Soil-Pile Interaction Diagram [modified from Abghari & Chai, 1995]

SIMULATION OF SHAKING TABLE TEST EXPERIMENTS

DESCRIPTION OF SHAKING TABLE EXPERIMENTS

A series of scaled physical model experiments have been performed at U.C. Berkeley on the shaking table to examine the seismic response of soil-pile-superstructure interaction, as described a the companion paper by Meymand et al. [2000]. A model soil with appropriately scaled stiffness and strength properties was developed for the project, and consisted of 72% kaolinite, 24% bentonite, and 4% type C fly ash (by weight). The model soil has a unit weight of 14.8 kN/m^3, a plasticity index of 75, an undrained shear strength of 4.8 kPa and a shear wave velocity of approximately 32 m/second (the last two parameters measured at a water content of 130% and cure time of 5 days). Bender element and cyclic triaxial laboratory tests were performed to characterize the modulus degradation and damping curves of the model soil. From standard Caltrans design, a 410 mm diameter x 12.7 mm wall concrete-filled steel pipe pile was selected as the target prototype. Scaling constraints dictated a maximum prototype pile length of 12.8 m, which provided a L/d ratio of 32, acceptable for a slender pile. The fixity conditions of the pile, known to be significant in lateral response, were established as fixed against rotation at the head, and fixed against (relative) translation at the tip. Meymand et al. [1998] provide a more detailed discussion of model soil development and pile selection.

The work presented here focuses on the analytical simulation of one shaking table experiment, referred to as Test 1.18. The layout of this experiment is shown in Figure 2 and consists of four single piles with head masses ranging from 10 to 160 lbs. in approximately 6 ft of cohesive model soil. The target prototype soil is San

Francisco Bay Mud, and the modeling criteria is almost exclusively based on the undrained shear strength and soil stiffness, which was found to be controlled by water content. As shown by the results of the T-bar pullout tests in Figure 3, a non-uniform soil strength profile was obtained [Meymand et al., 1998]. The model itself was subjected to a series of seismic events including sine sweeps and earthquake records. In particular, the scaled version (a_{max} = 0.35g) of the YBI90 motion recorded at Yerba Buena Island during the Loma Prieta earthquake was used as the input to the shaking table for the test analyzed herein. Figure 3 shows the location of the vertical arrays of accelerometers and T-bar tests. The figure also shows the approximate head masses and location for each of the piles.

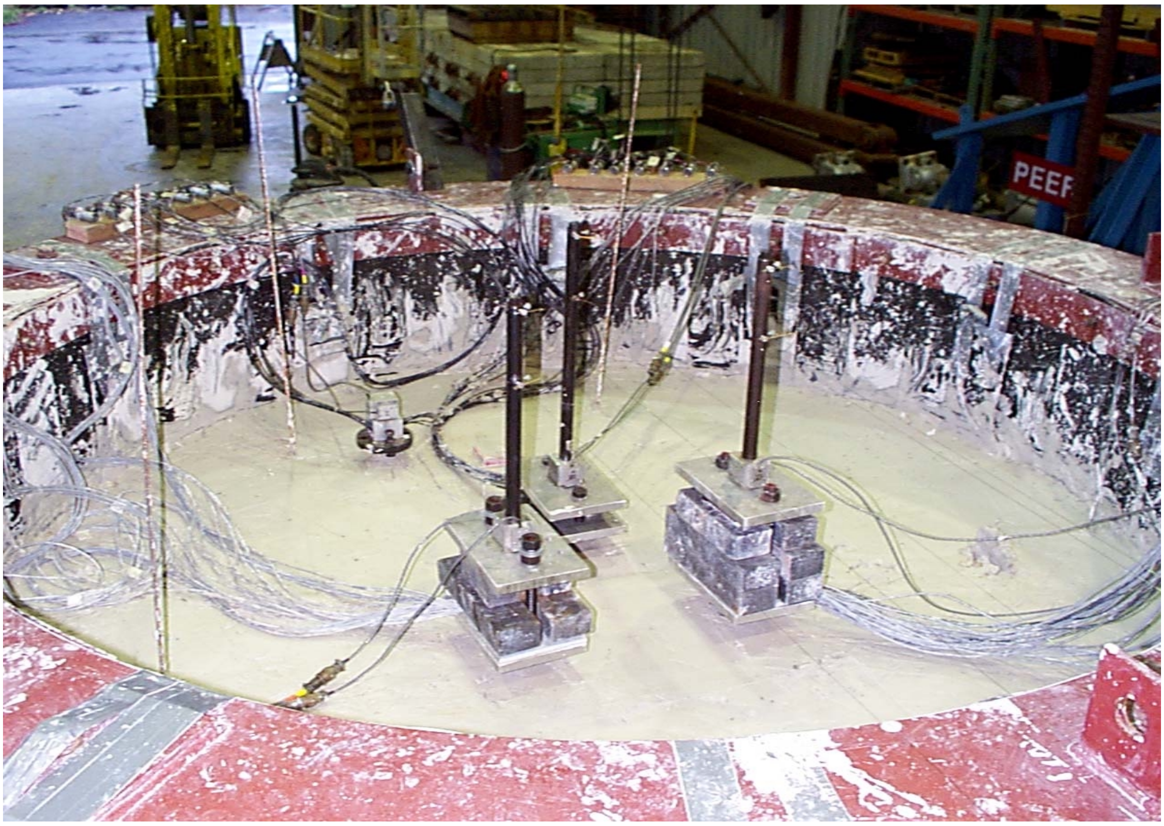
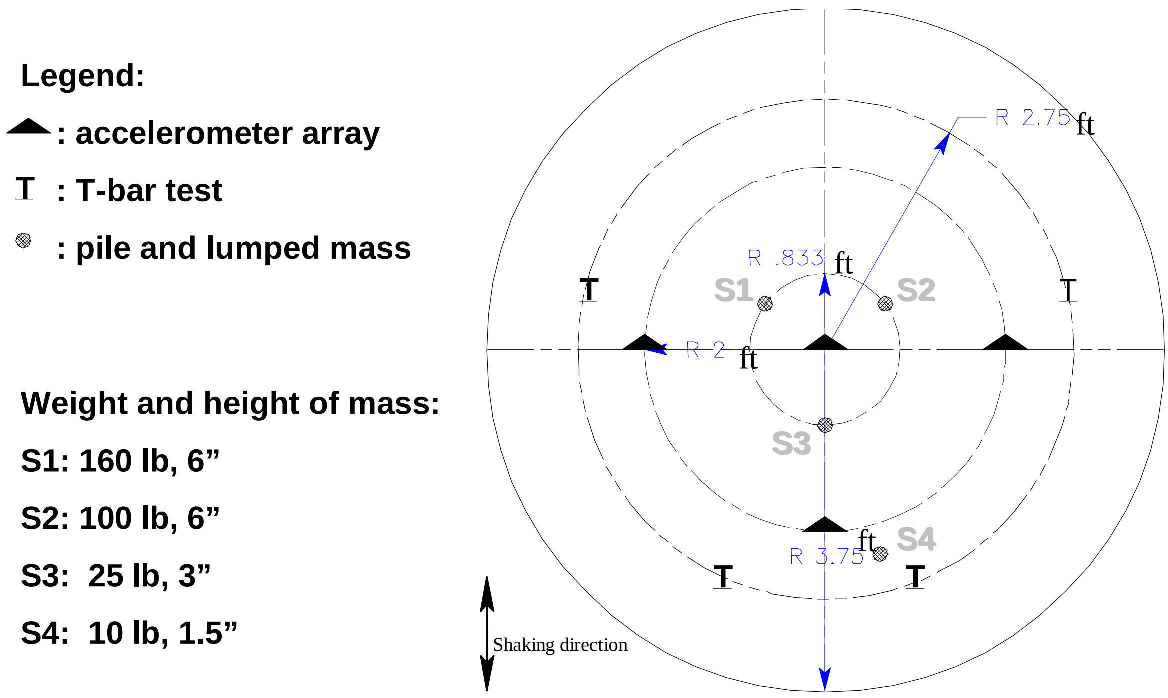


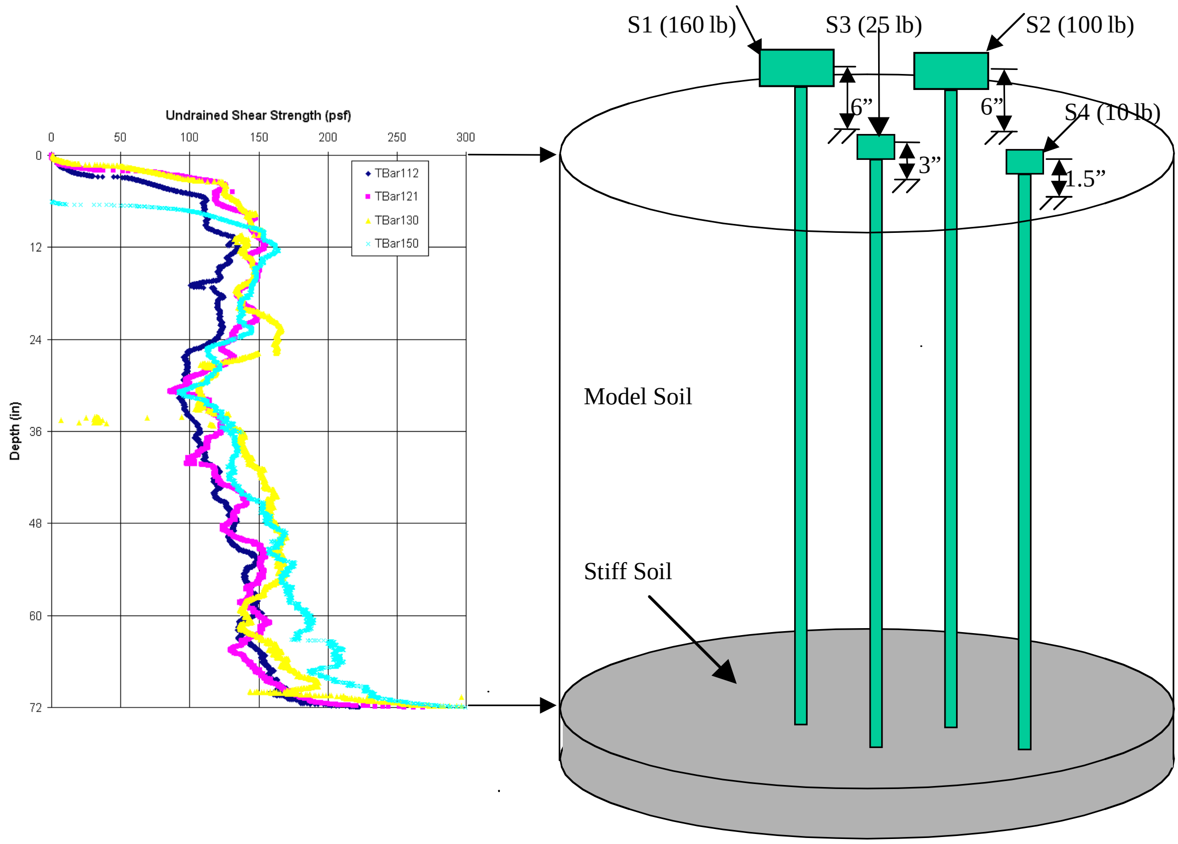
Figure 2: Test Arrangement in the Large Shaking Table

Free-Field Response

The free field response of the container was evaluated by comparing the motions recorded at two of the vertical arrays placed inside the container with those numerically simulated. The equivalent linear method of analysis in the time domain was used for describing the site response analysis. The model uses an equivalent Rayleigh damping formulation and the implementation is essentially similar to that of QUAD4M [Hudson et al., 1994]. Lok [1999] shows results suggesting that for this level of shaking the equivalent linear method as well as the use of other nonlinear constitutive laws give very similar results. In general, the frequency content of the spectral accelerations is well predicted except at frequencies higher than 5 Hz. In particular, the amplification of spectral acceleration at the site period (i.e., $T \approx 0.5$ sec.) was well captured by both linear and nonlinear analyses. The equivalent linear method gives better overall predictions for periods shorter than 0.2 sec. However, the spectral accelerations at frequencies higher than 5 Hz were underpredicted by up to 50% by both methods of analyses. Most significantly, the amplification of the spectral acceleration at about T=0.2 sec, corresponding to the predominant frequency of the earthquake motion, was underpredicted in the results of both simulations.



a) Plan View of Shaking Table Test 1.1



b) Shear Strength Profile and Set-up of Single Piles

Figure 3: Diagram of Shaking Table Test Series 1.1.

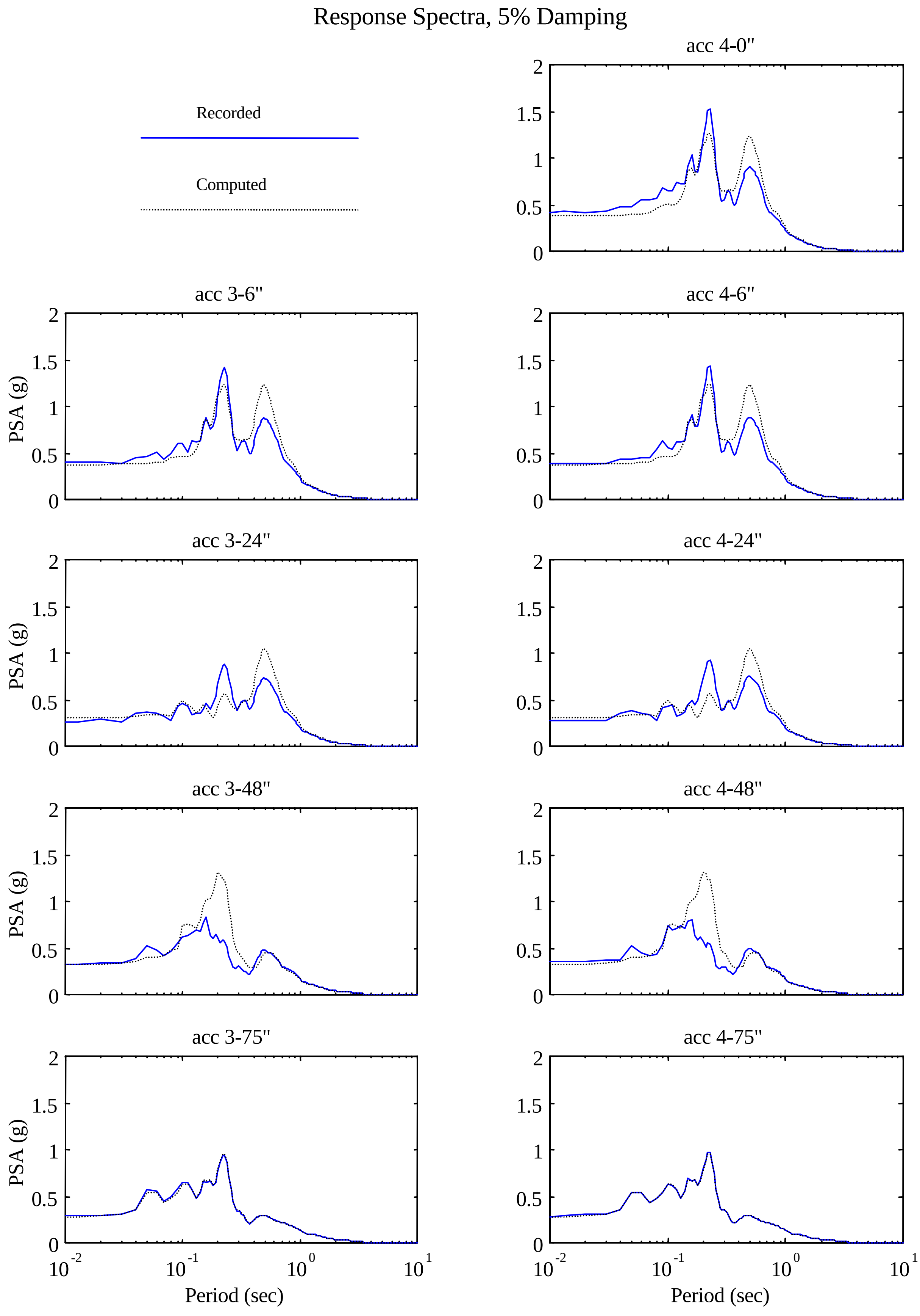


Figure 4: Comparison of Observed and Calculated Free Field Acceleration Response Spectra, Test 1.18.

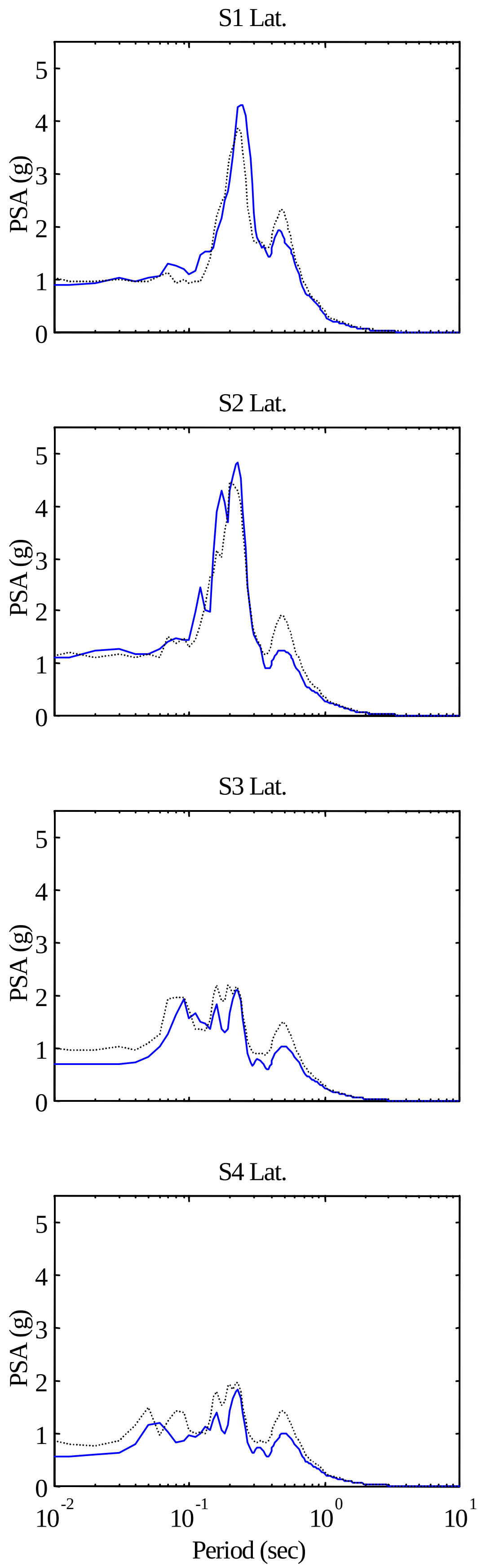


Figure 5: Comparison of Observed and Calculated Superstructure Response Spectra for Test 1.18

Structural Response

Figure 5 shows the acceleration response spectra of the four structures, respectively. In general, the computed response spectra of the four structures for Test 1.18 provide a reasonable match with the observed behavior. The frequency content of the structural response was well predicted for all the cases. The predicted responses for the two heavy structures, S1 and S2, match the observed spectral response very well. The predicted responses for the two small structures, S3 and S4, match less favorably with the observed response, which is partially attributed to the prediction of the free field response. Figure 6 compares the computed and recorded maximum accelerations. The predicted maximum accelerations of the structures are within 5-10% to the observed one except for the pile S3. In particular, the error for the two small structures, S3 and S4, are much higher than the two large structures, S1 and S2, which indicates that the accuracy of the structural prediction is dominated by the accuracy of the free field prediction. Therefore, greater error was observed for the smaller structures, which were essentially dominated by kinematic interaction. As the mass of the structure becomes larger, inertial interaction has greater influence to the overall response of the structure, and the influence of the free field accuracy is less significant.

CONCLUSIONS

The coupled dynamic soil-pile-superstructure and site response analysis using the finite element computer code GeoFEAP was presented. The computer code incorporates a 1-D element to model the nonlinear near field response simulated through p-y springs and a 2-D solid element using the equivalent linear approach to represent the free field site response. The responses of single piles were calculated and compared to the observed behavior in the large shaking table. Detailed comparisons show that GeoFEAP is capable of simulating the observed behavior of soil-pile-superstructure interaction and describes the transition from kinematic to inertial interaction. In particular, the predicted frequency content of the response spectra for the superstructures match most favorably with the experimental results. The overall magnitude and frequency content of the spectral accelerations were in good agreement. The coupled formulation is successful in providing results comparable to those from the more commonly used uncoupled formulation, and has the simplicity of solving the SPSI problem and the site response analysis in a single step.

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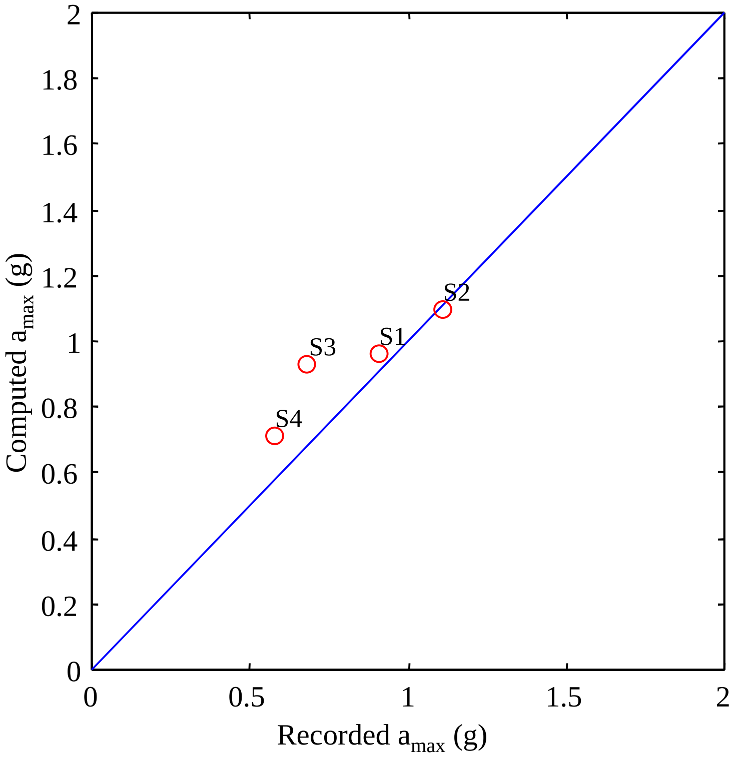

Figure 6: Computed a_{max} vs. Recorded a_{max}, Test 1.18

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